ARIZONA DEPARTMENT OF TRANSPORTATION

REPORT NUMBER: AZ-SP-9001

VARIABILITY IN ESTIMATION OF STRUCTURAL CAPACITY OF EXISTING PAVEMENTS FROM FALLING WEIGHT DEFLECTOMETER DATA

Special Report

Final Report

Prepared by:
A.S.M. Mustaque Hossain, Ph.D.
Arizona Transportation Research Center
206 S. 17th Avenue
Phoenix, Arizona 85007

John P. Zaniewski, Ph.D., P.E.
Center for Advanced Research in Transportation
College of Engineering & Applied Sciences
Arizona State University
Tempe, Arizona 85287-6306

August, 1990

Prepared for:
Arizona Department of Transportation
206 South 17th Avenue
Phoenix, Arizona 85007
in cooperation with
U.S. Department of Transportation
Federal Highway Administration

Technical Report Documentation Page

1. Report No.
AZ-SP-9001

2. Government Accession No.

3. Recipient's Catalog No.

4. Title and Subtitle
Variability in Estimation of Structural Capacity of Existing Pavements from Falling Weight Deflectometer Data

5. Report Date August, 1990

6. Performing Organization Code

7. Author (s) A.S.M. Mustaque Hossain, Ph.D.
John P. Zaniewski, Ph.D., P.E.

8. Performing Organization Report No.

9. Performing Organization Name and Address
Arizona Transportation Research Center &
Center for Advanced Research in Transportation
Arizona State University
Tempe, Arizona 85287-5306

10. Work Unit No.

11. Contract or Grant No.

12. Sponsoring Agency Name and Address
ARIZONA DEPARTMENT OF TRANSPORTATION
206 SOUTH 17TH AVENUE
PHOENIX, ARIZONA 85007

13. Type of Report & Period Covered

14. Sponsoring Agency Code

15. Supplementary Notes
Prepared in cooperation with the U.S. Department of Transportation, Federal Highway Administration

16 Abstract In this report, variability of the structural capacity estimation of existing pavements by the mechanistic method is presented. The analysis was performed to find the sensitivity of estimated structural capacity towards input Falling Weight Deflectometer (FWD) deflection data, backcalculated layer moduli, layer thicknesses, and the temperature correction factor for asphaltic layer moduli. FWD data collected on 16 in-service pavements in Arizona were used in the study.

Existing pavements show very high variability in their structural characteristics. The variation of sensor readings for a span of the pavement is the same for all the sensors. Deflection measurements on a long span of the road show more variability than short span. The variability in measured deflections magnify the variability of backcalculated layer moduli, i.e. small variability in measured deflections over a section of a pavement will result in large differences of backcalculated layer moduli, especially for asphalt concrete and base layers. This variation for either layer is the same over a span of the pavement regardless of span length. The number of FWD tests required to characterize the pavement structurally was found to be one on a short span (90 feet) and five on a longer span (one mile) of the pavement. Asphalt concrete, base and subbase moduli for a 5-layered pavement as well as their interactions were found to have significant effect on the structural capacity of the pavement.

The temperature correction factors for asphaltic layer moduli suggested in the AASHTO Guide are very sensitive to change in temperature and have large influence on the estimated structural capacity from fatigue analysis.

Depending on the thickness of the layers, 0.5 inch to 1 inch variation in asphalt concrete thickness and 1 inch to 2 inch variation in base thickness of 5-layered granular base pavements affect the calculation of structural capacity by the mechanistic method significantly. No significant effect was observed for up to 2 inches variation of subbase thickness.

17. Key Words
Deflection, Mechanistic, FWD, Sensor, Asphalt Concrete, Variability, ANOVA, Structural Capacity, Moduli.

18. Distribution Statement
Document is available to the U.S. Public through the National Technical Information Service, Springfield, Virginia, 22161

23. Registrant's Seal

19. Security Classification (of this report)
Unclassified

20. Security Classification (of this page)
Unclassified

21. No. of Pages

22. Price

TABLE OF CONTENTS

LIST OF TABLES

Table		Page

LIST OF FIGURES

INTRODUCTION

As the nation's highway infrastructure grows older, reconstruction and rehabilitation of the vast road network are increasingly important. Today, a major share of highway funds are dedicated to maintenance, rehabilitation and reconstruction of the pavements. Replacing and rehabilitating existing pavements around the nation requires an estimated four hundred billion dollars over the next fifteen years (1). The costs to the road users of poor pavements will be several times this amount. Thus, rational studies in pavement evaluation are needed to save nation's transportation dollars.

Pavement showing signs of structural deficiency require rehabilitation and strengthening. The structural capacity of the existing pavement system, or in other words remaining life, needs evaluation as part of the design process. Considerable savings in rehabilitation cost can be made by predicting the strength of the existing pavement system accurately and taking this into account in the design process.

The magnitude of stresses and strains in the pavement indicate the structural capacity of a pavement system. Flexible pavement responses correlating with the visual distresses are:

1) tensile strain at the bottom of asphalt concrete layer and
2) vertical compressive strain on the top of the subgrade (2).

These stress/strain parameters are calculated by the mechanistic analysis of pavements. The analysis requires the characteristics of the materials in the layers of the pavement as inputs. The material characterization can be can be done in two ways: 1) destructive testing coupled with laboratory testing and 2) non-destructive testing.

Destructive testing requires coring an existing pavement and laboratory characterization of the material properties. Tests methods include the resilient modulus, indirect tensile strength, fatigue resistance, R-value and the California Bearing Ratio (CBR). These results define an empirical measure of the structural capacity of the pavement. In-situ CBR and Plate Load tests can also be performed. Destructive evaluation has serious deficiencies. Coring is slow, expensive and causes lengthy disruption to traffic. Also, most of the laboratory test results are of empirical nature and do not correlate well with the in-situ structural properties of the material (3).

Moreover, since sampling and testing for each site requires considerable effort, time and money, only a few sites might be used to characterize several miles of a roadway.

Non-destructive testing (NDT) is now widely recognized as an important tool for pavement structural evaluation. With NDT, structural capacity of a pavement can be evaluated without disturbing the pavement. State of the art NDT evaluation measures a pavement's deflection response to a known load. The load generated by a NDT device may be static (Benkelman Beam), steady-state vibratory (Dynaflect and Road Rater) or impulse (Falling Weight Deflectometers). Though surface deflection data analysis is a matter of continuing research, non-destructive testing for measuring surface deflection is accepted by most highway agencies as a standard practice for the advantages of being fast and reliable in most of the cases. The new AASHTO Guide for Design of Pavement Structures (4) recommends the use of "dynamic" NDT deflection measuring devices for surface deflection measurements. Numerous field and laboratory investigations examined relationships between pavement performance and deflection (5, 6, 7, 8). With deflection testing, a through evaluation pavement response can be obtained by closely spacing test sites.

The deflections measured with NDT are used to estimate the moduli of pavement layers. The pavement is modelled by a suitable approach such as linear elastic theory, or linear or non-linear finite element methods. Moduli estimates are determined with a "backcalculation" technique. For the test load-pavement combination computed deflections are compared to measured deflections. The moduli of the layers are varied until the computed and measured deflections are approximately equal. The surface layer and other asphaltic layer moduli thus obtained are modified to take in to account the temperature at the time of testing. These moduli are then used to compute the effective structural capacity of the pavement according a pavement design procedure such as the AASHTO Guide (4). Figure 1 shows the use of deflection basin results to predict in-situ moduli for multiple layered pavement system.

The Falling Weight Deflectometer (FWD) employs a mass falling on to a buffered circular load plate. Developed in Europe, FWD's have become popular in the United States. The load pulse shape of FWD's simulates traffic loads better than other deflection devices (9,10). FWDs can transmit relatively heavy loads to the pavements compared with the other deflection testing devices. Usually the load range is 1,500 lbs. to 35,000 lbs. depending on the FWD model. The magnitude of the dropping mass and drop height are altered to change

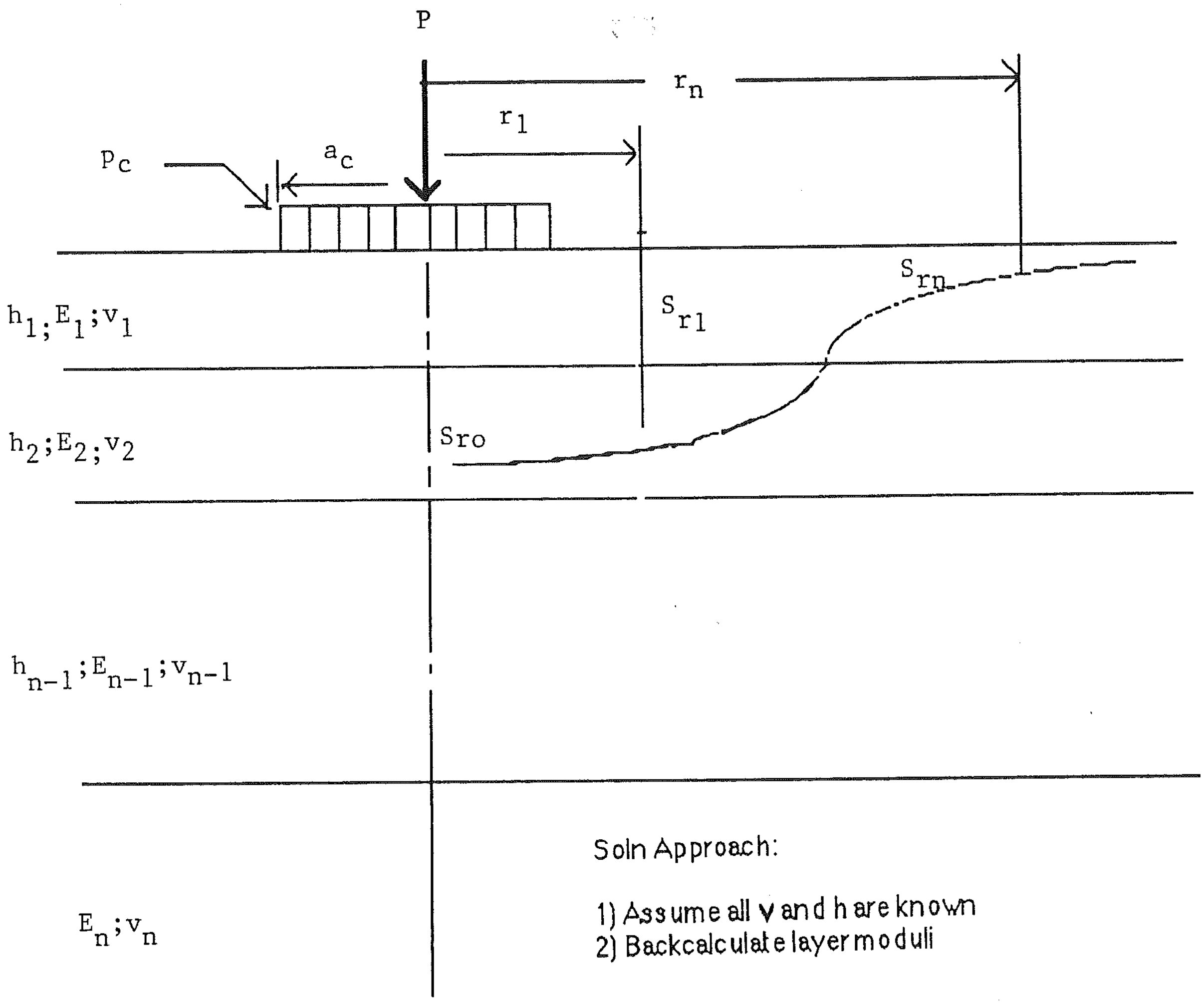


Figure 1- NDT Deflection Basin Analysis for Estimating In-situ Layer Moduli (After AASHTO (4))

the applied load levels.The FWD has small preload, 3 to 14% of the maximum load. The applied load is measured by a load cell. The load pulse is approximately of a half sine wave form with a duration of 30-40 milliseconds.

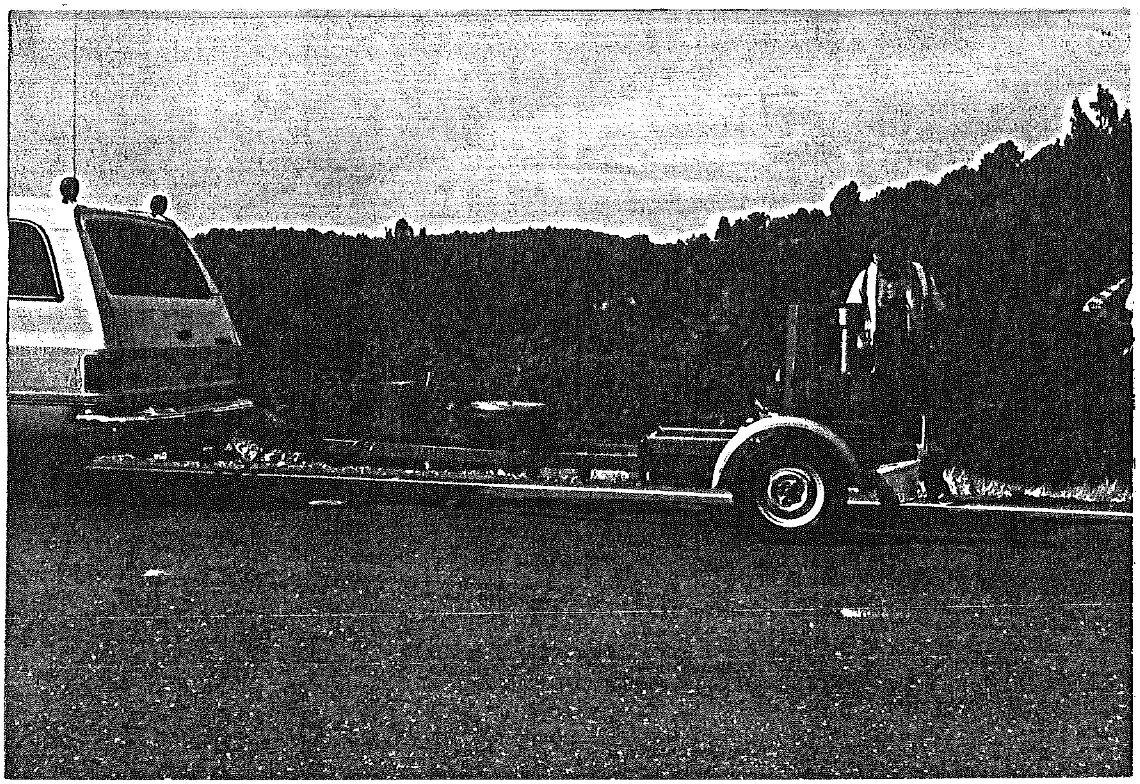
Velocity transducers measure the peak deflection under the load and at several locations away from the load. The sensors are mounted on a bar which is automatically lowered with the loading plate. Measured deflections can be plotted as deflection basins.

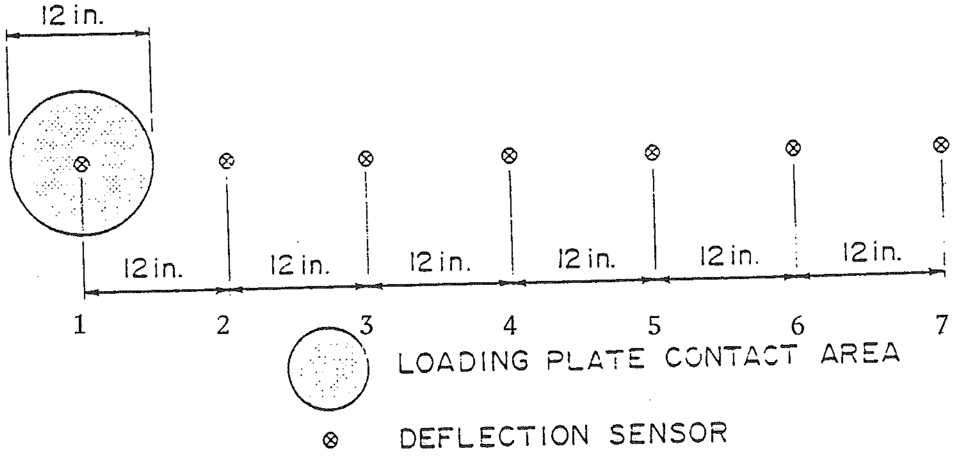
Arizona Department of Transportation (ADOT) purchased a Dynatest model 8000 FWD unit in 1982 and updated to 8002 unit in 1987. Figure 2 shows a Dynatest 8002 FWD and typical sensor setting used by ADOT. The operating sequence of Dynatest FWD is fully automated. Load is applied by a single falling mass. Factory calibrated geophones register the peak deflections due to an applied load. The load range is from 1,500 lbs. to 27,000 lbs.

As with any other NDT device, the FWD has a few minor drawbacks. Hoffman and Thompson (11) have shown that FWD deflection pulse is narrower than that of the moving wheel. Also, the ground accelerations of the moving wheel load are typically one-tenth of the FWD induced accelerations. As a result, the inertia of pavement mass affects response to FWD loads while inertia is negligible for the moving wheel load. However, Hoffman and Thompson concluded that deflections under wheel load and FWD compared very favorably. Despite these minor drawbacks, FWD is the best NDT deflection device available to date because of its fast testing, ability to simulate wheel load, ability to apply heavy loads and to measure a complete deflection basin.

PROBLEM DEFINITION

Calculating pavement structural capacity in terms of the ability to carry 18Kip Equivalent Single Axle Load (18KESAL) repetitions from Falling Weight Deflectometer (FWD) data is a three-step procedure. First, the layer moduli are backcalculated from the FWD, layer type and thickness data. Second, the critical pavement response, usually the tensile strain at the bottom of asphalt concrete layer, is calculated. Finally, empirical relationships are used for estimating the number of 18KESAL based on the critical pavement response. The relationship estimates the number of 18KESAL repetitions the pavement can carry before fatigue failure. Variability in any stage of the analysis affectsthe estimation of structural capacity by the mechanistic method.





12 in.
12 in.
12 in.
12 in.
12 in.
12 in.
12 in.
1
2
3
4
5
6
7
LOADING PLATE CONTACT AREA
DEFLECTION SENSOR

OBJECTIVES

The objectives of this study were:

1) To find the variability of calculated 18KESALs with respect to the variability in input FWD data and corresponding backcalculated layer moduli over short spans and long spans of pavement sections.

2) To find the number of FWD tests required to characterize the pavement structurally over a short span (90 feet) and a long span (1 mile).

3) To find the effect of the temperature correction factor for asphaltic layer moduli on the estimated structural capacity.

4) To find the coring needs to extract thickness information about pavement cross-section at FWD test points on existing pavements.

DATA COLLECTION

Table 1 lists the sixteen sites selected in this study, and Table 2 shows the pavement sections of these sites. Figure 3 shows the locations of the sites. All deflection data were collected with a Dynatest model 8002 FWD. As shown on Figure 2, the deflection sensors were spaced at 12 inch intervals with the first sensor located at the center of the load. The target load was 9000 lbs. At Sites 1 through 13 deflection were measured in the outer wheel path at 10 locations spaced at 10 foot intervals. For Sites 14 to 16, deflection locations were collected every 0.1 mile.

ANALYSIS METHOD

The analysis process consists of i) backcalculation of layer moduli of the pavements from FWD data and ii) computation of structural capacity of the existing pavement through fatigue analysis. Backcalculation of layer moduli was done with the Arizona Deflection Analysis Method (ADAM) developed by Hossain (12). ADAM uses the CHEVRON (13, 14) computer program for pavement response analysis. A robust optimization routine iterates the moduli values to minimize the squared error between the measured and calculated deflection basins. The backcalculated layer moduli were used to determine the tensile strain at the

TABLE 1- LOCATION OF TEST SITES AND PAVEMENT TYPES

Site	Location	Route	Mile Post	Pavement Type	Test Type
1	Benson	I10W	300.07	5-layer	10 tests/90 ft
2	Winslow	I40E	260.21	4-layer	10 tests/90 ft
3	Minnetonka	I40E	261.78	4-layer	10 tests/90 ft.
4	Dead River	I40E	317.06	4-layer	10 tests/90 ft.
5	Flagstaff	I17N	337.00	4-layer	10 tests/90 ft.
6	Crazy Creek	I40E	323.78	4-layer	10 tests/90 ft.
7	Sunset Point	I17N	251.41	5-layer	10 tests/90 ft.
8	Seligman	I40W	131.71	4-layer	10 tests/90 ft.
9	Benson East	I10W	303.00	4-layer	10 tests/90 ft.
10	Jacob Lake	US89AN	578.00	4-layer	10 tests/90 ft.
11	Morristown	US60W	120.00	4-layer	10 tests/90 ft
12	McNary	US260E	369.00	5-layer	10 tests/90 ft
13	Kingman I	I40E	59.00	4-layer	10 tests/90 ft
14	Yucca	I40W	33.00	4-layer	10 tests/mile
15	Kingman II	I40E	24.00	4-layer	10 tests/mile
16	Tombstone	U80E	316.50	4-layer	10 tests/mile

TABLE 2- LAYER TYPE AND THICKNESS AT DIFFERENT SITES

Site/	Layer 1		Layer 2		Layer 3		Layer 4		Layer 5	
Sta	Mat	Thk (in)	Mat	Thk (in)	Mat	Thk (in)	Mat	Thk (in)	Mat	Thk (in)
1/1	AC	7	BS	2.5	AB	2	SB	12	SC-SM*	
2/1	AC	12	BTB	3	SB	5	SM*	-	-	-
3/1	AC	11.5	BTB	2	SB	3	SM*	-	-	-
4/1	AC	8	CTB	4.5	SB	7	SM*	-	-	-
5/1	AC	9	AB	4	SB	12	-	-	-	-
6/1	AC	8	CTB	6	SB	6	SM*	-	-	-
7/1	AC	6	BS	4	SB	26	SGS	6	CL-CH*	-
8/1	AC	6	AB	6	SB	24	CH*	-	-	-
9/1	AC	6	AB	6	SB	18	SC-SM* -	-	-	
10/1	AC	9	BS	4	AB	4	SC-CH* -	-	-	
11/1	AC	4.25	AB	4	SB	15	-	-	-	-
12/1	AC	4.8	BS	2.2	AB	3	SB	6	-	-
13/1	AC	9.5	AB	4	SB	15	-	-	-	-
14/1	AC	4.0	AB	4	SB	9	-	-	-	-
15/1	AC	4.0	AB	4	SB	9	-	-	-	-
16/1	AC	3.0	AB	4	SB	15	-	-	-	-

* Subgrade Classification based on Unified Method.

Note: AC: Asphalt Concrete, BS: Bituminous Surface, BTB: Bituminous Treated Base, CTB: Cement Treated Base, AB: Aggregate Base, SGS: Subgrade Seal, SB: Sub Base (Select Material)

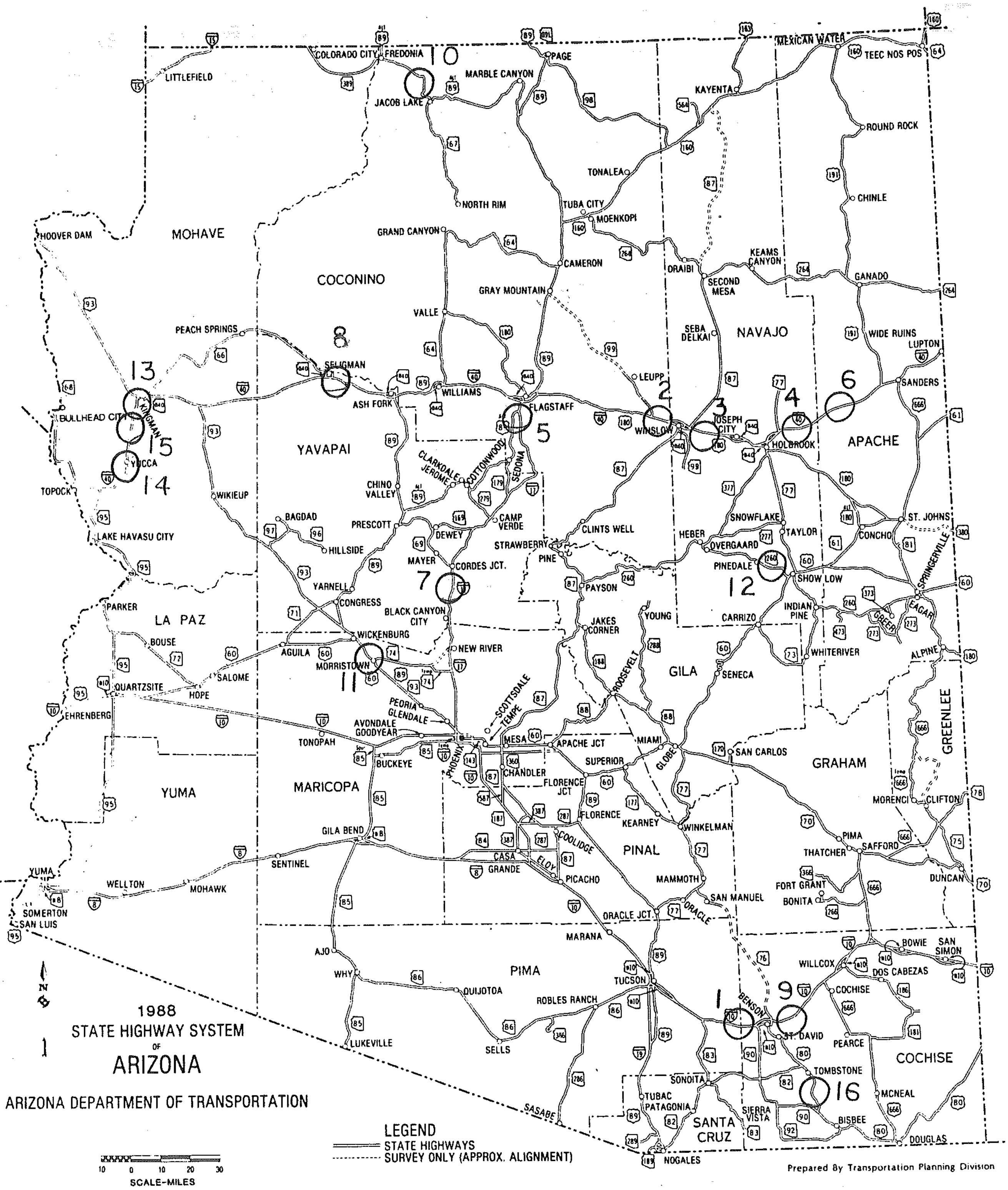


Figure 3 Location of Test Sites

bottom of the asphalt concrete layer. The structural capacity of the pavement in terms of theoretical number of 18KESALs was determined using the following equation for fatigue analysis developed by Hossain (12):

$$N = (2.265 \times 10^{-7})\,(1/e_{ac})^{3.84} \qquad .....(1)$$

where

N = theoretical number of 18Kip ESAL repetitions to fatigue failure

e_{ac} = tensile strain at the bottom of asphalt concrete layer (micro inch/inch)

Figure 4 shows the flow chart of the analysis process.

SPATIAL VARIABILITY OF FWD DEFLECTION DATA

The structural capacity of a pavement is affected by the spatial variability of the measured deflections. Variability is the result of equipment repeatability and spatial characteristics of the pavement structure and materials. Mamlouk et al. (3) concluded equipment variability is insignificant compared to spatial variability.

In this section, the spatial variability of FWD deflection data across 90 feet span of 13 sites and across one mile span of 3 sites is presented. Table A.1 in the Appendix A shows the variation in sensor readings for the sites listed in Table 1. The coefficients of variation for all the sensors varies from 2.80% to 41.7% for 90-foot span and 27% to 57% for 1-mile span. Variations of sensor readings are higher for 1-mile span of the road than for the short road span. This spatial variability of deflection measurements reflects the variability of the structural response of the existing pavement sections along the roadway.

Table 3 show the average coefficient of variation for each sensor for pavements with granular and stabilized bases. The average coefficient of variation is almost the same for every sensor, i.e., variability in response measured by the first sensor is similar to the response measured by the seventh sensor.

Pavements with bituminous surface (BS) layer have the highest average coefficient of variation for all the sensors for 90-foot span sites. The pavements with cement and bituminous treated base layers showed the lowest variation in measured deflections. These stabilized bases may be responsible for the uniform response of the pavements to the applied FWD load.

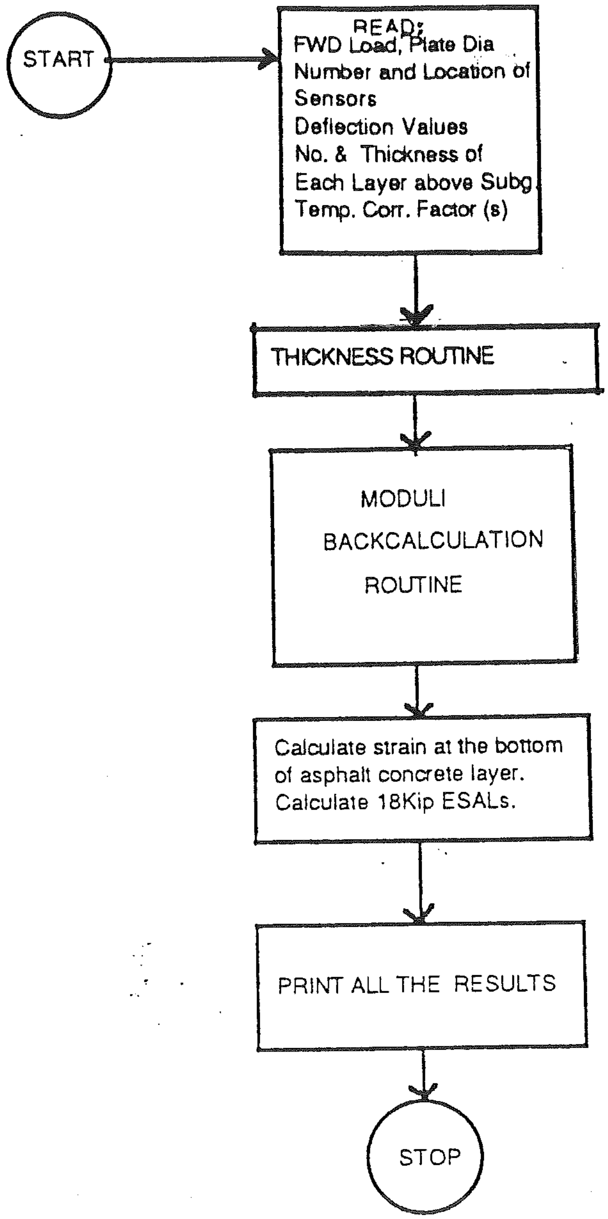


Figure 4- Flow Chart of the Analysis Process

TABLE 3- SPATIAL VARIABILITY OF DEFLECTION MEASUREMENTS

	Coefficient of Variation (%)							
Base Type	Avg. for Sensor No.							Avg. all Sensors
	1	2	3	4	5	6	7	
AB	14.2	12.1	11.6	11.1	10.5	11.8	14.4	12.2
AB (long span)	31.6	34.5	38.9	41.1	43.0	42.2	41.6	39.0
BS	15.5	14.9	12.8	13.6	15.5	18.3	18.9	15.6
BTB	11.9	11.8	10.2	10.3	9.3	9.0	9.0	10.2
CTB	10.7	10.6	8.9	8.8	6.8	8.0	9.0	9.0

Note: AC: Asphalt Concrete, BS: Bituminous Surface, BTB: Bituminous Treated Base, CTB: Cement Treated Base, AB: Aggregate Base

As shown in Table 4 spatial variability for different sites was compared with several factors including:

1. the surface condition expressed in terms of percent cracking,
2. the coefficients of variation of backcalculated asphalt concrete moduli over the section and
3. calculated theoretical number of 18Kip Equivalent Single Axle Loads (18KESALs) the sections can carry before fatigue failure

The percent cracking data were extracted from the ADOT pavement management system inventory database. These crack data are for 1,000 square feet at the milepost location rather than for the entire pavement area. The validity of these data with respect to the pavement condition at the point where the deflection measurements were made is questionable. Variability in deflection measurements over a span of the road cannot be explained by the distress condition on the surface and may depend on the other factors, such as subgrade type and moisture content and properties of other layers.

Since the deflection measurements are used to compute moduli and subsequently number of allowable applications, variability in deflection measurements will produce variability in the computed parameters.

However, as shown in Table 4, the variability of the computed parameters actually increases at each step in the process. In every case, the coefficient of variability of the asphalt modulus is greater than for the measured deflections. The coefficient of variability of the computed allowable axle loads is greater than the variability in the moduli values.

TABLE 4- COMPARISION OF SPATIAL VARIABILITY OF MEASURED DEFLECTIONS WITH ESTIMATED STRUCTURAL CAPACITY

Base Type	Average Cracking (%)	Avg. C.V. All Sensors (%)	Avg. C.V EAC (%)	Avg. C.V. Calculated N_{18} (%)
AB	2.40	12.2	28.0	69.7
AB (long span)	20.3	39.0	22.3	57.7
BS	0.25	15.6	31.0	56.4
BTB	0.00	10.2	20.0	47.3
CTB	0.00	9.00	31.0	60.4

Note: AC: Asphalt Concrete, BS: Bituminous Surface, BTB: Bituminous Treated Base, CTB: Cement Treated Base, AB: Aggregate Base

EFFECT OF VARIATION IN LAYER MODULI ON STRUCTURAL CAPACITY

The variability in the backcalculated layer moduli affects the estimated structural capacity. In order to study the effect of layer moduli on the estimated structural capacity, the factorial design shown in Figure 5 with the levels of layer moduli are shown in Table 5 was analyzed. Five levels of AC (surface), AB (aggregate base) and SM (select material/subbase) modulus were selected for each of the three pre-defined pavement categories: weak, medium and stiff. Figure 6 shows the cross-section for each type of pavement. The CHEVRON program (13, 14) was run for each of the $3 \times 5^3 = 625$ pavements. Tensile strain at the bottom of the asphalt concrete layer was calculated for a 9,000 lb. wheel load and 100 psi tire pressure.

E_{AC} (ksi)	1					2					3					4					5				
E_{AB} (ksi)	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
E_{SM} (ksi)																									
1																									
2																									
3																									
4																									
5																									

Figure 5- Factorial to Study the Effect of Layer Moduli on the Structural Capacity of the Pavements

TABLE 5- LEVELS OF LAYER MODULI USED TO STUDY MODULUS VARIABILITY EFFECT

Pavement Type	Layer Type	Modulus (ksi) at Level 1	2	3	4	5
STIFF	AC	325	488	650	819	975
	AB	20	30	40	50	60
	SM	12.5	19	25	32.5	37.5
MEDIUM	AC	225	337.5	450	562.5	675
	AB	15	22.5	30	37.5	35
	SM	10	15	20	25	30
WEAK	AC	125	187.5	250	312.5	375
	AB	10	15	20	25	30
	SM	5	7.5	10	12.5	15

Note: AC: Asphalt Concrete, AB: Aggregate Base, SM: Select Material

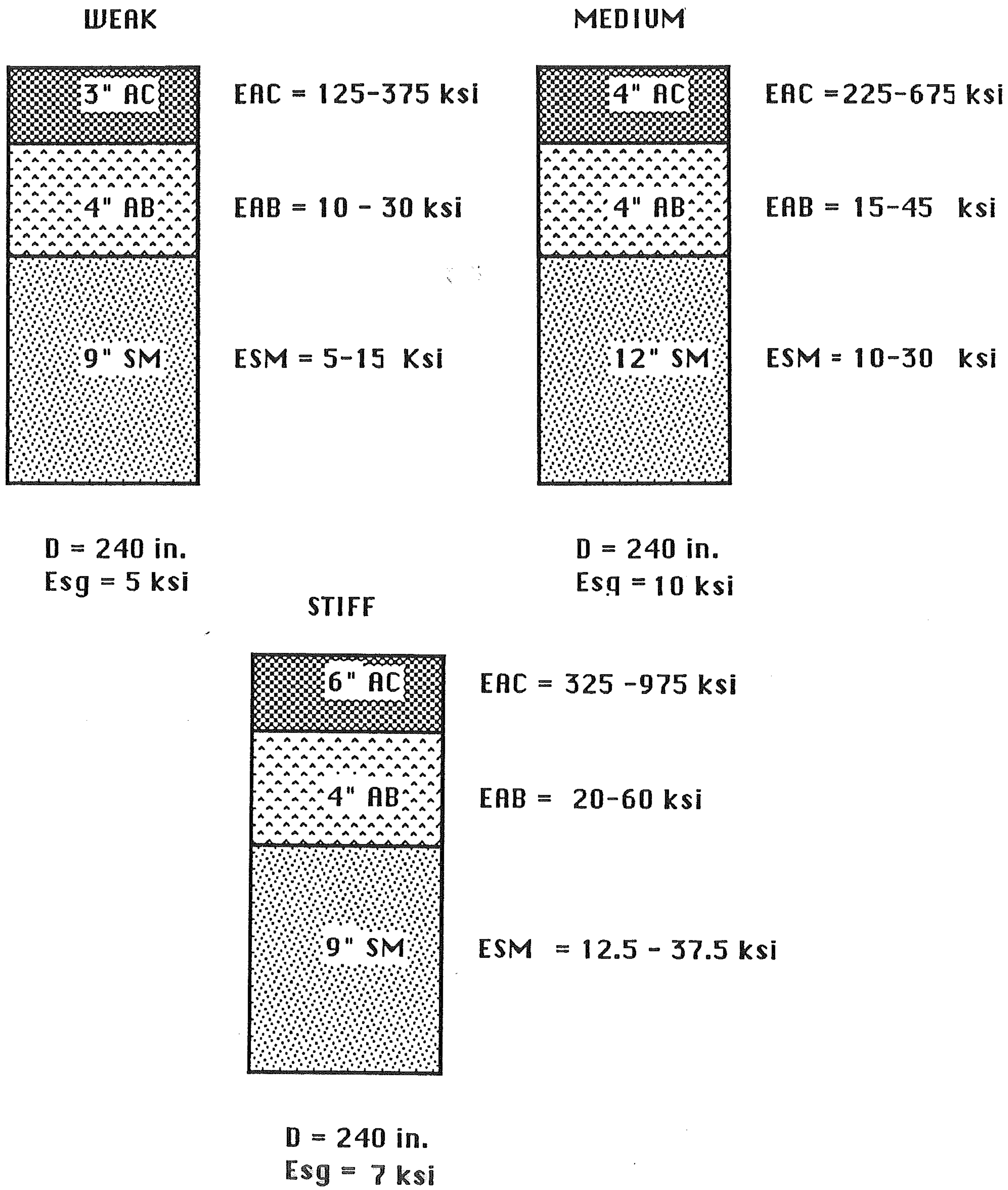


Figure 6- Cross Sections for Different Pavement Types

The tensile strain was substituted into Equation 1 to estimate the number of 18KESALs the pavement can carry before fatigue failure. Tables A.2, A.3 and A.4 in the Appendix A give the correlation between layer moduli and 18KESAL for weak, medium stiff and stiff pavements respectively. Significance of correlations were evaluated with the Student t-tests.

The moduli of all the layers are significantly correlated with the estimated 18KESALs for each pavement type. The correlation coefficients are higher than 0.5 for AC and AB layer. For the weak pavement, AB modulus appears to have a stronger effect than AC modulus on the calculated 18KESALs whereas for medium and stiff pavement, AC modulus appears to affect the calculation of 18KESALs more than any other layer moduli. To study the effect of interaction of layer moduli, the factorial in Figure 6 was developed for medium stiff type of pavement.

The Analysis of Variance (ANOVA) was used to describe the variation in calculated 18KESALs with the following variance components:

Source of Variation	Definition	Type of Effect
ACE	Modulus of AC Layer	Fixed
ABE	Modulus of AB Layer	Fixed
SME	Modulus of SM Layer	Fixed
ACE*ABE	Interaction of Moduli of AC and AB Layers	
ACE*SME	Interaction of Moduli of AC and SM Layers	
ABE*SME	Interaction of Moduli of AB and SM Layers	
ACE*ABE*SME	Interaction of Moduli of AC, AB and SM Layers	

The following model was proposed:

$$N18_{ijk} = \mu + ACE_i + ABE_j + SME_k + ACEABE_{ij} + ACESME_{ik} + ABESME_{jk} + ACEABESME_{ijk} + \epsilon_{(ijk)}$$

$i = 1,....,3$

E_{AC} (ksi)	337.5			450.0			562.5		
E_{AB} (ksi)	22.5	30.0	37.5	22.5	30.0	37.5	22.5	30.0	37.5
E_{SM} (ksi)									
15									
20									
25									

Figure 7- Factorial for Medium Stiff Type of Pavement

$j = 1,....,3$

$k = 1,....,3$

.....(2)

where:

$N18_{ijk}$ = the theoretical 18KESALs calculated at the i^{th} level of AC modulus, the j^{th} level of AB modulus and at the k^{th} level of SM modulus.

μ = overall mean

ACE_i = the effect of the i^{th} level of (fixed) treatment AC Modulus.

ABE_j = the effect of the j^{th} level of (fixed) treatment AB modulus.

SME_k = the effect of the k^{th} level of (fixed) treatment AB modulus.

$ACEABE_{ij}$ = the interaction effect between the i^{th} level of AC modulus and the j^{th} level of AB modulus.

$ACESME_{ik}$ = the interaction effect between the i^{th} level of AC modulus and the k^{th} level of SM modulus.

$ABESME_{jk}$ = the interaction effect between the j^{th} level of AB modulus and the k^{th} level of SM modulus.

$ACEABESME_{(ijk)}$ = the interaction effect between the i^{th} level of AC modulus, the j^{th} level of AB modulus and the k^{th} level of SM modulus.

$\epsilon_{(ijk)}$ = the (random) within error. The $\epsilon_{(ijk)}$ s are assumed to be normally and independently distributed with mean zero and variance σ^2.

It is important to note that in model (2), the subscripts for ACEABESME and ϵ are the same, indicating the effects due to interaction of all the layer moduli and the error are confounded. This is necessary because of absence of any replication. Table 6 shows the ANOVA for this factorial.

From the table, all the main factors or layer moduli and two factor interactions (or interaction between two layer moduli) were significant at 5%. The interaction between layer moduli, which was used as the error term, might be significant. Thus, the model described in Equation (2) is not adequate to capture all the variation. However, physical interpretation of response of the flexible pavement system to the applied load also supports the assertion that not only the layer moduli, but also the interactions between the layer moduli dictate the structural response of a multi-layer system. Further research should examine the precision needed for

TABLE 6 - ANOVA FOR MEDIUM STIFF TYPE OF PAVEMENT

Source of Variation	Sum of Squares	Degrees of Freedom	Mean Squares	Fo
ACE	63.0	2	31.5	6300*
ABE	24.9	2	12.45	2494*
SME	8.49	2	4.25	848*
ACE*ABE	1.63	4	0.41	82*
ABE*SME	0.54	4	0.14	28*
ACE*SME	1.09	4	0.27	54*
ACE*ABE*SME/ Error	0.04	8	0.005	
Total	99.72	26	-	

* Significant at $\alpha = 5\%$

resilient moduli test of pavement materials to determine the effect of test variance on estimated structural capacity.

In the backcalculation of layer moduli, variation in estimated layer modulus from actual values is compensated by the variations in the moduli of other layers in the structure, providing a "compensating effect". The resultant capacity of the pavement system with backcalculated layer moduli remains essentially unchanged (15). The significant interactions of layer moduli in the ANOVA analysis supports the compensating effect concept in the backcalculation analysis.

SPATIAL VARIABILITY OF BACKCALCULATED LAYER MODULI

Deflection data from each station of the sites listed in Table 1 were analyzed to define the spatial variability of the backcalculated layer moduli. Table A.5 in the Appendix A shows the mean, standard deviation and coefficient of variation of the layer moduli. The average coefficient of variation for layer moduli for the short span sections varies from 15% to 46% with a mean value of 29%. The average coefficient of variation for layer moduli for three sites with ten deflection tests per mile varies from 22% to 34% with a mean value of 28%. Interestingly the average coefficient of variation for the two data sets is almost equal.

The variation in AC and base (AB, BS, BTB or CTB) moduli are quite remarkable. The average coefficient of variation of AC modulus for nine sites is 28% whereas for base modulus the average coefficient of variation is 36%. These variations strongly affect the structural capacity determined from fatigue analysis. Interaction of high asphalt concrete modulus and base modulus tends to give very low asphalt concrete tensile strain and consequently a very high number of 18KESAL applications. The opposite is true for interaction of low asphalt concrete modulus and low base modulus. As a result, the computed structural capacity is highly variable.

Table 7 compares average coefficient of variation of layer moduli to the estimated structural capacity for the 1-mile span sites listed in Table 1. Variation of asphalt concrete and base moduli is highly magnified in the calculation of structural capacity.

TABLE 7- EFFECT OF VARIATION OF BACKCALCULTED LAYER MODULI ON THE ESTIMATED STRUCTURAL CAPACITY

Site	Average Coefficient of Variation (%)				
	E_{AC}	E_{AB}	E_{SM}	E_{SG}	N_{18}
14	13	10	54	10	23
15	19	16	42	40	44
16	35	40	39	20	106

DETERMINATION OF REQUIRED NUMBER OF FWD TESTS FOR A PROJECT

Researchers differ on the issue of required number of FWD Tests needed for structural characterization of pavements. The AASHTO Guide for the Design of Pavement Structures (4) recommends a spacing of 300 to 500 feet when accurate historic data for a section are unavailable. When accurate historic data are available, the Guide recommends 10 to 15 test points per mile. No analysis was presented in the Guide supporting this recommendation. ARE Inc. (16) recommended Dynaflect tests every 100 feet when the subgrade is nonuniform. For uniform subgrade, the spacing can be extended to 250 feet. Karan et al. (17) used a spacing of 6 deflection tests per kilometer (roughly 10 per mile). Koole (18) proposed a spacing of 66 feet for an overlay design method. ADOT studied the variability of Dynaflect deflection data and concluded that one measurement per mile is required for network level pavement management system (19). Shell Research (20) recommends one FWD test per 85 to 165 feet. Lytton et al. (21) concluded that a minimum of 5 tests per mile are required at the network level to rank pavement sections. Project level evaluation requires one test every 100-300 feet in each wheel path.

In this study, testing frequency was determined by statistical analysis of existing FWD data for the pavement sections listed in Table 1. The statistical technique for determining the sample size of any response parameter is straight forward. If a response parameter is normally distributed, then for small sample size, the required number of samples to have certain statistical confidence on the mean of the response parameter can be found as:

$$n = (t_{\alpha/2} \times s)^2 / H^2 \qquad .....(3)$$

where:

H = half width of the confidence interval

$t_{\alpha/2}$ = critical value of t from the t-distribution at certain degrees of freedom and at level of significance α.

s = standard deviation computed from the samples, and

n = required sample size.

For FWD tests on existing pavements, the response parameter may be the deflection measured by the first sensor normalized to a standard load, backcalculated asphalt concrete modulus, or the calculated structural capacity of the pavement in terms of 18KESALs.

From Equation 3, it is obvious for calculating the required number of tests to satisfy some statistical criteria, the designer needs to quantify three parameters:

1) half width of confidence interval on the response parameter, H which is representative of estimation of error or allowable deviation from the mean value,
2) standard deviation of the response parameter computed from the sample data, and
3) the t-value at a certain level of significance, α.

The designer must select α. It should be noted that t-values can be determined from the standard t-table corresponding to a certain o and degrees of freedom, v. The degrees of freedom equals to n-1 where n is the required sample size. Since n is not known beforehand, the solution must be done by trial and error. In general, decreasing the width of confidence interval increases the number of samples required.

For specific projects estimates of s and H are not available before the testing. An option may be to collect some data on the project and estimate s and H from that sample and to perform additional testing if necessary. However, this must be done for each project since every project will have different standard deviation for any response parameter selected. This process is statistically correct for determining the required number of tests, however, it is not feasible for a highway agency because of time and cost involved in the process. The calculations can be automated and performed by the FWD computer, but going back over the section a second time creates traffic control problems. If it can be shown that the standard deviation of the response parameter is constant for all the roads in the network, then a uniform sampling rate can be used for the entire network.

Number of Tests Required Over a Short Span

Of the projects listed in Table 1, FWD data collected on 9 projects were analyzed to determine the number of FWD tests required over 90-foot span of the pavement. The number of tests were sorted in the following fixed fashion:

NO. OF TESTS	LOCATION
10	At the beginning of the project and at 10 feet interval
5	At the beginning and at 20 feet interval.
3	At the beginning and at 30 feet interval.
1	At the beginning of the project.

It was assumed that 10 FWD tests per 90 feet represented the standard or truth for this particular experiment. It is also inherently assumed that the samples are a random selection from the population of pavements.

The backcalculated asphalt concrete modulus and the structural capacity in terms of estimated number of 18KESALs the sections can carry were selected as the response parameters. Tables B.1 and B.2 in Appendix B give the mean and standard deviation for the backcalculated AC modulus and the estimated number of 18KESALs the pavements can carry before fatigue failure for each of the nine sites for each sample size. Table 8 summarizes the mean, pooled standard deviation and coefficient of variation for the above parameters for each sample size.

The size of the coefficients of variation for the backcalculated asphalt concrete moduli are quite remarkable and much smaller than anticipated. Linear regressions were conducted between the mean asphalt concrete moduli values and 18KESALs derived by taking different sample sizes. Table B.3 in the Appendix B lists the parameters derived from the linear regression analysis.

TABLE 8- SUMMARY STATISTICS OF BACKCALCULATED AC MODULUS AND STRUCUTURAL CAPACITY CORRESPONDING TO DIFFERENT TESTING FREQUENCY

No. of Tests per Site	N	E_{AC} (ksi)	N_{18} (millions)
		MEAN	
10	9	222	41.7
5	9	221	42.8
3	9	212	42.9
1	9	234	42.4
		POOLED STANDARD DEVIATION OF THE GROUP	
10	9	48.3	65.2
5	9	57.2	17.0
3	9	55.4	21.8
		COEFFICIENT OF VARIATION	
10	9	21.7	156.3
5	9	25.9	39.70
3	9	26.1	50.80

Figures 8, 9 and 10 illustrate the linear regression between the mean backcalculated asphalt concrete moduli.

The coefficients of determination, R^2 for the linear regression between one test per 90 feet and average of ten tests per 90 feet for nine projects are 0.79 and 0.94 for backcalculated AC modulus and 18KESALs respectively. It appears that one FWD test per 90 feet very closely approximates the average of ten tests per 90 feet for both asphalt concrete modulus and structural capacity.

The assumed linear relationship between backcalculated asphalt concrete modulus and 18KESALs from one FWD test per 90 feet and from ten FWD tests per mile was verified by the analysis of variance (ANOVA). The relationship appeared to be significant. Also, paired t-tests were conducted between the values of these parameters from one test per 90 feet and ten tests per 90 feet. No significant difference was detected at 5% level of significance. So, one test could be done to characterize the pavement structurally over a shorter span of the road up to 90 feet.

Number of Tests Required Over A Longer Span

FWD deflection measurements were taken at the beginning of the project and at nine locations at a uniform interval of 0.1 mile for the sites 14 through 16 listed in Table 1. To determine the required testing frequency, 7, 5, 3 tests were randomly selected out of 10 tests. The current ADOT practice is to take three tests per mile.

The FWD data were used to backcalculate the layer moduli and to estimate the number of 18KESALs each pavement can carry before fatigue failure. Table B.6 in the Appendix B shows the mean, standard deviation and coefficient of variation of the number of 18KESALs (N_{18}) for the three sites. Kolmogorov-Smirnov (K-S) (22) test verified the data are normally distributed. The coefficient of variation is nearly equal for 7 and 5 tests per mile. However, for 3 tests per mile the variation is very high.

Student's t-tests were conducted to detect the difference in means of N_{18} calculated at each site corresponding to different testing frequency. No significant difference was detected between means of 18KESALs calculated from 10, 7, 5 or 3 tests. It may be noted that because of the high standard deviation

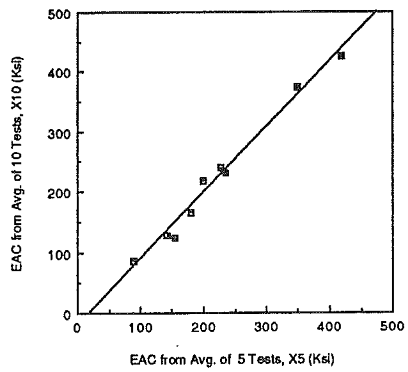


Figure 8- Average of Ten Tests Versus Average of Five Tests

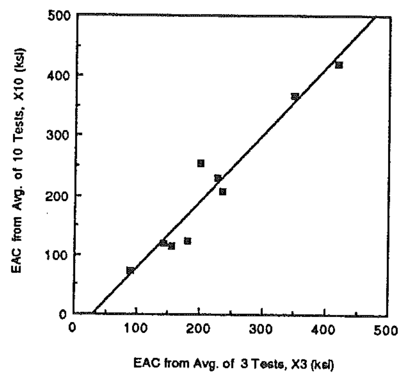


Figure 9- Average of Ten Tests Versus Average of Three Tests

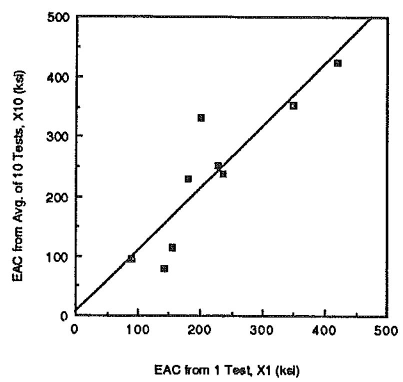


Figure 10- Average of Ten Tests Versus One Test

associated with each mean, the pooled standard deviation during the t-tests is also high. This high standard deviation is responsible for the low t-statistic during mean testing.

The results of the t-test suggest that 3 tests per mile are as good as 10 tests per mile to characterize the pavement structurally. However, the coefficients of variation of 18KESALs for 3 tests per mile appear to fluctuate widely compared to those for 5 tests per mile. The average coefficient of variation of 18KESALs corresponding to 5 tests per mile and 3 tests per mile for all three runs at each location is shown in Table 9. It appears that the average coefficients of determination for the calculated 18KESALs for 3 tests per mile are similar to those for 5 tests per mile.

Student's t-tests were also done between means of 18KESALs from 7, 5 and 3 tests per mile. Significant difference was observed between means of 18KESALs computed from 5 and 3 test results for a single run for Site 15. However, in two other runs for Site 15, no difference was detected.

Differences between 3 tests per mile and 5 tests per mile further investigated using the deflection basin parameter A obtained by fitting an exponential curve of the form $Y = A^{*}e^{BX}$. Where Y is the deflection in mils and X is the radial distance, to the deflection basin normalized to a 9,000 lb. load. Table 10 shows the means, standard deviation and coefficient of variation of parameter A for different sites. It is clear that the coefficient of variation of parameter A is much less than that for calculated 18KESALs for every site.

Kolmogorov - Smirnov test results shown in Table B.10 in the Appendix B demonstrate the basin parameters are normally distributed. Student's t-tests were then conducted between the means of A to detect any significant differences between testing frequency. There is no significant difference between means of A by taking 10, 7, 5 or 3 tests per mile except between 5 tests per mile and 3 tests per mile for Site 15.

This limited study suggests 5 tests per mile would be a viable choice for a project level decision on structural capacity estimation for the existing pavements. The failure of the Student's t test to detect a statistically significant difference between 3 and 5 tests per mile is due to the variability associated with 3 tests per mile. Hence, this level of testing was rejected in favor of 5 tests per mile. A larger number of sections should be studied to further support or reject conclusion.

TABLE 9- COEFFICIENT OF VARIATION OF ESTIMATED STRUCTURAL CAPACITY CORRESPONDING TO DIFFERENT TESTING FREQUENCY

Location	Type of Test	No. of Tests per Mile	Average Coefficient of Variation (%)
14	Random	5	42
		3	43
15	Random	5	24
		3	17
16	Random	5	66
		3	61

TABLE 10- EFFECT OF TESTING FREQUENCY ON BASIN PARAMETER A

Site	No. of Tests per mile	Basin Parameter, A Mean	Std. Dev.	C.V. (%)
14	10	9.04	1.11	12.0
	7	8.98	1.09	12.0
	5	8.93	1.15	13.0
	3	9.20	1.47	16.0
15	10	12.68	0.89	7.0
	7	12.41	0.74	6.0
	5	13.00	0.82	6.0
	3	11.92	0.40	3.0
16	10	9.976	0.59	6.0
	7	10.12	0.61	6.0
	5	10.00	0.55	5.5
	3	9.846	0.89	9.0

EFFECT OF TEMPERATURE CORRECTION FACTOR ON COMPUTED STRUCTURAL CAPACITY

Asphalt properties, especially the modulus of elasticity, are highly dependent on temperature. Since modulus affects the deflection measurements, the modulus of the layers which are temperature dependent (like, AC, BS and BTB) must be corrected to a standard temperature, usually 70^{O}F (4). The AASHTO Design Guide (4) has a graph for temperature corrections of the asphaltic layer moduli to this standardized temperature based on:

1. the air temperature at the time of FWD testing
2. five day mean air temperature before the testing date and
3. thickness of the asphalt bound layer.

The backcalculated asphalt concrete or asphalt treated base layer moduli are multiplied by these factors. These adjusted moduli can be used for determination of structural layer coefficient from the nomographs in AASHTO Design Guide (4).

In mechanistic analysis, the moduli are used as inputs for critical response calculation in fatigue analysis. Since there is no limit on the value of layer moduli, the resulting structural capacity analysis from fatigue criterion could result in a very high number of 18KESAL repetitions. This is particularly true when the temperature at the time of the test is greater than the reference temperature resulting in an upward adjustment of the asphalt modulus. At high modulus values there are low strains calculated corresponding to a stiff asphaltic layer.

In order to study the effect of temperature correction factor on the estimated structural capacity by mechanistic analysis developed in this study, the pavement section in Site 16 was evaluated. The pavement temperature of this site was calculated from the nomograph in AASHTO Guide (4) corresponding to air temperature at the time of deflection testing plus 5-day mean air temperature before deflection testing and thickness of asphalt concrete layer. The temperature adjustment factor of 2.5 corresponding to the pavement temperature was determined from the nomograph in the guide. The backcalculated asphalt concrete layer modulus was corrected with this factor and six other factors which are derived by varying the actual temperature correction factor by $\pm 25\%$, $\pm 50\%$ and $\pm 75\%$. The corresponding temperature correction factors were 4.375, 3.75, 3.125, 1.875, 1.25 and 0.625. The theoretical number of 18KESALs calculated corresponding to the temperature adjusted moduli are shown in Table 11.

A close inspection of temperature correction factor nomograph shows that the correction factor is very sensitive to the changes in temperature, thus making the surface modulus very sensitive to test temperature. Consequently, the calculated structural capacity of the pavement becomes very sensitive to temperature factor. By varying the temperature correction factor from -75% to +75%, the estimated 18KESALs vary from -69% to +153% percent.

In order to find a relationship between the 18KESALs and temperature adjustment factor, Fc for the pavement in Site 16 an exponential curve of the following form was fitted:

$$N_{18} = 1.315 \text{ X } e^{0.568 \text{ x } Fc} \qquad R^2 = 0.996$$

$$n = 7 \quad SEE = 0.055$$

From the previous relation it is obvious that if the temperature correction factor changes by a tenth of a unit, the calculated 18KESALs change by 0.08 millions for the pavement in Site 16. The temperature correction factor is a very sensitive parameter, especially in the calculation of 18KESALs. It is apparent that the temperature correction factors for asphaltic layer moduli are major sources of variation in structural capacity estimation of pavements by the mechanistic method. It is questionable at this time, whether this factor is a very good adjustment parameter for asphalt concrete modulus to represent the field condition especially when the pavement temperature is very high (greater than 130^{o}F).

EFFECT OF VARIATION IN LAYER THICKNESS ON STRUCTURAL CAPACITY

Pavement layer thickness is a primary input of all backcalculation procedures. Thickness data can be obtained either from construction data or cores taken from the pavement. These thicknesses may vary from the design thickness because of variability in the construction process. Also, existing pavements may receive treatments, such as asphalt concrete friction course, which are not expected to increase the structural capacity but contribute to the total thickness of the AC layer. Little information is available in the literature about the effect of thickness variation on the estimated structural capacity from FWD deflection data. Rwebangira et al. (23) concluded that variation of asphalt concrete and base layer thicknesses affect the backcalculated layer

TABLE 11- EFFECT OF TEMPERATURE CORRECTION FACTOR ON ESTIMATED STRUCTURAL CAPACITY

Factor	Var. (%)	Temp. (^oF)	EAC (ksi)	Diff (%)	18KESALs (millions)	Diff (%)
4.375	+75	101	897	+75	14.7	+153
3.750	+50	98	780	+50	11.1	+91
3.125	+25	94	650	+25	8.2	+41
2.50*	0.0	88	520	0.0	5.8	0.0
1.875	-25	83	390	-25	3.9	-33
1.250	-50	80	260	-50	2.6	-55
0.625	-75	62	130	-75	1.8	-69

* Actual value

Note: Uncorrected asphalt concrete modulus = 208 ksi moduli. However, the backcalculated layer moduli are more sensitive to asphalt concrete thickness than base layer thickness. Irwin et al. (24) showed that random deflection measurement errors combined with random variability of pavement layer thickness can lead to a high degree of "pseudo-variability" in the backcalculated layer moduli. They recommend accurate determination of layer thicknesses in order to reduce the inaccuracy of the resultant backcalculated layer moduli.

To study the effect of layer thickness on the calculated structural capacity of the pavements, sites 5, 9, and 11 from Table 1 were selected. The thickness for the AC layer for these sites ranges from 4.3 inches to 9.0 inches whereas the AB layer thickness ranges from 4.0 inches to 6.0 inches. The subbase layer thickness ranges from 12.0 inches to 18.0 inches. Two sets of deflection basins from ten stations within each site were analyzed for each site representing deflection basins with the highest and lowest first sensor deflections (normalized to 9,000 lbs).

The experiment was designed to capture the effect of layer thickness on the response parameter or the theoretical structural capacity (expressed in terms of 18KESALs) of the pavement section from backcalculated layer moduli and fatigue analysis. Figure 11 shows the full factorial designed to capture the effect of variability of layer thickness on the calculated structural capacity of the site 9. The thickness of the AC layer was varied by ± 0.5, ± 0.75 and ± 1.0 inch. The thickness of the AB and SM layers were varied by ± 0.5, ± 1.0, and ± 2.0 inches. Thus, a 7^3 factorial was designed for this site.

Analysis of Variance (ANOVA) was used to describe the variation in calculated 18KESALs with the following variance components:

Source of Variation	Definition
ACT	Thickness of AC Layer
ABBT	Thickness of AB Layer
SMT	Thickness of SM Layer
ACT*ABT	Interaction of Thickness of AC and AB Layers
ACT*SMT	Interaction of Thickness of AC and SM Layers
ABT*SMT	Interaction of Thickness of AB and SM Layers

TSM (in) \ TAC (in)	5.0	5.5	5.75	6.0	6.25	6.5	7.0
16							
17							
17.5							
18.0							
18.5							
19.0							
20.0							

Figure 11- Factorial to Study the Effect of Layer Thickness for Site 9

ACT*ABT*SMT — Interaction of Thickness of AC, AB and SM Layers

The following model was proposed for site 9:

$$N18_{ijkl} = \mu + ACT_i + ABT_j + SMT_k + ACTABT_{ij} + ACTSMT_{ik} + ABTSMT_{jk} + ACTABTSMT_{ijk} + \epsilon_{(ijkl)}$$

$i = 1,....,7$

$j = 1,....,7$

$k = 1,....,7$

$l = 1,2$

...(4)

where:

$N18_{ijkl}$ = the theoretical 18KESALs calculated from the l^{th} deflection basin at the i^{th} level of AC thickness, the j^{th} level of AB thickness and at the k^{th} level of SM thickness.

μ = overall mean

ACT_i = the effect of the i^{th} level of (fixed) treatment AC thickness.

ABT_j = the effect of the j^{th} level of (fixed) treatment AB thickness.

SMT_k = the effect of the k^{th} level of (fixed) treatment AB thickness.

$ACTABT_{ij}$ = the interaction effect between the ith level of AC thickness and the j^{th} level of AB thickness.

$ACTSMT_{ik}$ = the interaction effect between the i^{th} level of AC thickness and the k^{th} level of SM thickness.

$ABTSMT_{jk}$ = the interaction effect between the j^{th} level of AB thickness and the k^{th} level of SM thickness.

$ACTABTSMT_{ijk}$ = the interaction effect between the i^{th} level of AC thickness, the j^{th} level of AB thickness and the k^{th} level of SM thickness.

$\epsilon_{(ijkl)}$ = the (random) within error. The $\epsilon_{(ijkl)}$ s are assumed to be normally and independently distributed with mean zero and variance σ^2.

TABLE 12- ANOVA FOR SITE 9

Source of Variation	Sum of Squares	Degrees of Freedom	Mean Squares	Fo
ACT	4.9E14	6	8.1E13	4.3*
ABT	3.8E14	6	6.3E13	3.3*
SMT	6.1E12	6	1.0E12	0.05
ACT*ABT	1.4E14	36	3.9E12	0.20
ACT*SMT	2.4E13	36	6.5E11	0.03
ABT*SMT	3.4E13	36	9.6E11	0.05
ACT*ABT*SMT	2.9E14	216	1.3E12	0.07
Error	6.6E15	343	1.9E13	
Total	7.96E15	685		

* Significant at $\alpha = 5\%$

It is important to note that the replicate of deflection basins used in the backcalculation of layer moduli made it possible to estimate the error the in the model. Table 12 shows the ANOVA table for site 9. From this table, it is evident that different levels of AC and AB thickness result in significantly different structural capacity for site 9.

The quality of the estimate of variation of structural capacity for each thickness level depend on the stability of the variation in structural capacity across thickness combinations. This depends on how much the estimate of structural capacity variance calculated from the two deflection basins for each thickness combination fluctuates. To test this statistically, Bartlett's Test for constant variance was applied. The Bartlett's Chi-square statistic was found to be 353. The critical value applicable to this statistic at 5% level of significance is approximately 124. So the hypothesis of homogeneity of variance assumed in this test was rejected. Transformation of 18KESAL data was necessary to make the variances stable. The correlation coefficient between means of N_{18} for each thickness combination and the corresponding variance was +0.89. The suggested transformation in this case of positive correlation between mean and variance is the square root of original data (22). This transformation was applied to N_{18} data and Bartlett's test was repeated. The test statistic for the transformed data was 125 which is significant at 5% but insignificant at 2.5%. Anderson and McLean (25) state that the F-test used in the analysis of variance is robust against minor deviation from homogeneity of variance, thus the square root transformation appeared to be appropriate.

The ANOVA was repeated for the transformed data as shown in Table 13. The degrees of freedom for the error due to the transformation applied to the data (22). It is clear that levels of thickness of AC and AB layers significantly affect the structural capacity estimated by the mechanistic method.

In order to find out the levels of AC and AB thickness which are significantly different from each other, Duncan's Multiple Range Test (22) was applied to the means of 18KESALs corresponding to different levels of AC and AB thickness.

Figure 11 illustrates the results. The means which do not share a common underline are statistically different. It is evident from the results that if thickness for AC layer is decreased by more than 1 inch, then the corresponding calculated 18KESALs are significantly different but, if it is increased by 1 inch, the calculated 18KESALs remain statistically the same.

TABLE 13- ANOVA OF TRANSFORMED DATA FOR SITE 9

Source of Variation	Sum of Squares	Degrees of Freedom	Mean Squares	Fo
ACT	1.2E7	6	2.0E6	2.5*
ABT	1.1E7	6	1.9E6	2.4*
SMT	0.4E6	6	.06E6	0.07
ACT*ABT	3.2E7	36	.09E6	0.11
ACT*SMT	0.9E6	36	.02E6	0.03
ABT*SMT	1.3E7	36	.04E6	0.05
ACT*ABT*SMT	1.03E7	216	.05E6	0.06
Error	2.7E8	342	.08E6	
Total	3.5E8	685		

* Significant at α = 5%

AC:

$N18^1$	$N18^2$	$N18^3$	$N18^4$	$N18^5$	$N18^6$	$N18^7$
(5)	(5.5)	(5.75)	(6.0*)	(6.25)	(6.5)	(7.0)

AB:

$N18^1$	$N18^2$	$N18^3$	$N18^4$	$N18^5$	$N18^6$	$N18^7$
(4.0)	(5.0)	(5.5)	(6.0*)	(6.5)	(7.0)	(8.0)

* Control Thickness

Note: Figures in parentheses are thickness in inches.

Figure 12- Duncan's Multiple Range Test for Means of 18KESALs for Site 9

For AB, a 2-inch deviation from the control thickness of 6 inch AB does not affect the calculation of 18KESALs. However, there is significant difference between means of 18KESALs computed from 5 inch AB and 7 inch AB. Thus, this site needs only knowledge of AC thickness within 0.5 inch of actual thickness to estimate the structural capacity.

Based on the analysis for site 9, the number of levels for layer thickness for sites 5 and 11 were decreased by 2 and the factorial was redefined to have five levels of thickness. The layer thickness for AC layer for both sites were varied by ± 0.5 inch and ± 1.0 inch while for AB and SM layers the thickness were varied by ± 1.0 and ± 2.0 inch. Thus, factorial with 5^3 combinations was designed for Sites 5 and 10 as illustrated in Figures 13 and 14.

The model assumed for analysis of variance was similar to one for site 9. Two deflection basins from a 90-foot span of each site were selected which correspond to maximum and minimum first sensor deflections normalized to a 9,000 lb load. The ANOVA tables for the 18KESALs computed from these deflections basins corresponding to $5^3 = 125$ combinations of layer thicknesses are shown in Table 14.

From the tables, it is clear that only AC and AB thickness significantly affect the structural capacity calculation. The homogeneity of variances in each of different layer thickness combinations for these sites were checked using Bartlett's test. Data for site 5 had a Bartlett's Chi-square statistic of 116 whereas the statistic for site 11 was 55. The critical value of this statistic at 5% level of significance for the data from both sites was 124. So, the variances for the 18KESALs computed for these sites were homogeneous and no transformation of data was necessary.

In order to determine the levels of AC and AB thickness which predicted significant different 18KESALs, Duncan's Multiple Range Test was applied. Figure 14 shows the test results for both sites. The means of 18KESALs corresponding to different levels of AC and AB thickness which do not share a common underline were found to be statistically different. For site 5, an AC thickness of 8" produced significantly different 18KESALs when compared to other levels of thickness. Here also, a decrease in 1 inch of AC thickness from the control thickness (9 inch) resulted in significantly different 18KESALs whereas an increase of 1 inch of AC thickness did not significantly affect the calculated 18KESALs. For AB, a 2 inch deviation from

TAC (in)	8.0					8.5					9.0					9.5					10.0				
TAB (in)	2	3	4	5	6	2	3	4	5	6	2	3	4	5	6	2	3	4	5	6	2	3	4	5	6
TSM (in)																									
13.0																									
14.0																									
15.0																									
16.0																									
17.0																									

Figure 13- Factorial for Site 5

TAC (in)	3.3					3.8					4.3					4.8					5.3				
TAB (in) / TSM (in)	2	3	4	5	6	2	3	4	5	6	2	3	4	5	6	2	3	4	5	6	2	3	4	5	6
10.0																									
11.0																									
12.0																									
13.0																									
14.0																									

Figure 14- Factorial for Site 11

TABLE 14- ANOVA FOR SITES 5 AND 11

SITE 5

Source of Variation	Sum of Squares	Degrees of Freedom	Mean Squares	Fo
ACT	2.90E17	4	7.2E16	6.5*
ABT	1.20E17	4	3.0E16	2.7*
SMT	8.50E16	4	2.1E16	1.90
ACT*ABT	1.50E17	16	9.5E15	0.86
ACT*SMT	9.70E16	16	6.1E15	0.55
ABT*SMT	1.30E16	16	7.9E14	0.07
ACT*ABT*SMT	6.80E16	64	1.1EE15	0.10
Error	1.40E18	125	1.1E16	
Total	2.23E18	249		

* Significant at $\alpha = 5\%$

SITE 11

Source of Variation	Sum of Squares	Degrees of Freedom	Mean Squares	Fo
ACT	6.48E15	4	1.62E15	7.7*
ABT	4.70E15	4	1.20E15	5.7*
SMT	7.20E13	4	1.79E13	0.09
ACT*ABT	2.64E15	16	1.65E14	0.77
ACT*SMT	4.38E14	16	2.74E13	0.13
ABT*SMT	2.40E14	16	1.49E13	0.07
ACT*ABT*SMT	1.70E15	64	2.64E13	0.13
Error	2.62E16	125	2.10E14	
Total	3.25E16	249		

* Significant at $\alpha = 5\%$

SITE 5

AC:

$N18^{1}$	$N18^{2}$	$N18^{3}$	$N18^{4}$	$N18^{5}$
(8.0)	(8.5)	(9.0*)	(9.5)	(10.0)

AB:

$N18^{1}$	$N18^{2}$	$N18^{3}$	$N18^{4}$	$N18^{5}$
(2.0)	(3.0)	(4.0*)	(5.0)	(6.0)

SITE 11

AC:

$N18^{1}$	$N18^{2}$	$N18^{3}$	$N18^{4}$	$N18^{5}$
(3.3)	(3.8)	(4.3*)	(4.8)	(5.3)

AB:

$N18^{1}$	$N18^{2}$	$N18^{3}$	$N18^{4}$	$N18^{5}$
(2.0)	(3.0)	(4.0*)	(5.0)	(6.0)

* Control thickness

Note: Figures in parentheses are thickness in inches.

Figure 15- Duncan's Multiple Range Test for Means of 18KESALs for Sites 5 and 11

a control thickness of 4 inches yielded significantly different 18KESALs, where as overestimation of thickness by 2 inch did not affect the calculation.

For Site 11, an AC thickness of 3.3 inch, which is 1 inch less than the control thickness of 4.3 inches, gave significantly different 18KESALs whereas 5.3 inches of AC did not yield significantly different 18KESALs. There was significant difference between the 18KESALs computed for 3.8 inch of AC and 3.3 inches of AC implying that 0.5 inch decrease in AC thickness for pavements having 3.8 inch or less AC thickness will produce significantly different 18KESALs. For AB thickness, an 1 inch decrease from the control thickness of 4 inch resulted in significantly different 18KESALs whereas overestimation of thickness by 2 inches did not produce any significantly different 18KESALs than from control thickness.

From the results of ANOVA analysis of these sites it is clear that for pavements with AC thickness of 4.0 to 9.0 inches, a 1 inch decrease in AC thickness will produce significantly different 18KESALs. For thin pavements, a 1 inch decrease in base thickness would result in different 18KESALs whereas for thick pavements, the calculated 18KESALs may or may not be affected by a 2-inch decrease in base layers depending on thickness of the AC layers. These results support the greater effect of base layer thickness on the calculation of 18KESALs for thin pavements as outlined earlier. Again, the overestimation of thickness of these layers above the actual layer thickness for which deflection test results are available do not affect the calculated 18KESALs.

If construction records for quality control show that as built thickness of AC and AB layers are varying more than 1 inch, then coring will be necessary to have accurate thickness of AC and AB layers at the FWD test locations for pavements having AC thickness of 4.0 to less than 6.0 inches and AB thickness of 4.0 inches. For pavements having AC thickness less than 4.0 inch, only 0.5 inch deviation from the mean value of AC thickness can be allowed and variation of AB thickness should be less than 1 inch. However, the base thickness variation may be more than 1 inch for thick pavements having AC thickness of 6 inches or more.

CONCLUSIONS

In this report, the variability of the structural capacity determination by the mechanistic methods was presented with respect to FWD input deflection data in the backcalculation scheme, backcalculated layer moduli and layer thicknesses and the temperature correction factor for asphaltic layer moduli.

The variability of deflection data has been found to be same for all the sensors. As can be expected, sensor readings on a longer span of the road show more variability than short span.

The variability in sensor readings are magnified when backcalculating the layer moduli, i.e. small variability in sensor data over a section of a pavement will result in high variability of calculated layer moduli. However, this variability is the same irrespective of the span of the roadway over which deflection testing is done.

All the layer moduli and their interaction affect the calculated structural capacity.

To characterize the pavement structurally, only one FWD test is required over a 90 feet span of the road and five FWD tests per mile are required.

The temperature correction factor suggested in the AASHTO Guide for correcting asphalt bound layer moduli were found to be very sensitive to temperature of the pavement and has tremendous effect on the estimated structural capacity of the pavement.

The variability in thickness of asphalt concrete and base layers also affects the estimated structural capacity but their interaction is not significant. This happens because of the fact that in a deflection matching backcalculation scheme, the thickness variation is compensated by a corresponding increase or decrease in modulus. For pavements with AC thickness of 4 inches or more, input thickness in FWD data analysis should not vary from the actual thickness by more than 1 inch whereas for pavements with less than 4 inches of AC thickness, the AC thickness should be known within 0.5 inch. The aggregate base thickness for such pavements should be known within less than 1.0 inch of actual thickness. For thick pavements with AC thickness of 6 inches or more, base thickness should be known within less than 2.0 inches of actual thickness. Thus, coring of FWD test locations is necessary for only AC layers for thick pavements, and AC and base layers for thin pavements.

RECOMMENDATIONS

The temperature correction factors from the AASHTO Guide for asphaltic layer moduli should be studied in detail to find a better correlation between temperature and in-situ layer moduli.

The suggested required number of FWD tests per mile to characterize the pavements structurally should be verified using data from more projects.

The thickness sensitivity analysis should be extended to include more projects with granular base and of asphalt concrete layer thickness different from those used in this study. The findings will supplement the suggested coring requirements for pavements with a wide range of asphalt concrete thickness. The study should also include pavements with stabilized bases so that coring needs for extracting thickness information for this type of pavements can also be addressed.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the financial support of this study provided by the Arizona Transportation Research Center, Arizona Department of Transportation. The computing facilities especially CRAY supercomputing facilities were provided by the Arizona State University. Thanks are due to Mr. Larry Scofield and Mr. John Eisenberg of ADOT and Dr. Dwayne Rollier and Mr. Rohan Perera of Arizona State University for their help in this study.

REFERENCES

1. SHRP, Strategic Highway Research Program. Participant Workbook, National Workshop, Austin, Texas, 1985.

2. Yoder, E.J. and Witczak, M.W.. Principles of Pavement Design, 2nd Ed., Wiley, New York, N.Y., 1975.

3. Mamlouk, M.S, Houston, W.N., Houston, S.L. and Zaniewski, J.P. Rational Characterization of Pavement Structures Using Deflection Analysis, Report No. FHWA-AZ88-254, Vol.1, Arizona Dept. of Transporatation, Phoenix, Arizona, 1988.

4. Lister, N.W. and Kennedy, C.K.. "A System for Prediction of Pavement Life and Design of Pavement Strengthening." Proc. 4th Intl. Confc. on Structural Design of Asphalt Pavements, Vol. 1, University of Michigan, Ann Arbor, 1977, 629-648.

5. Majidzadeh, K. and Ilves, G. Flexible Pavement Overlay Design Procedures, Vols. I & II, Report No. FHWA/RD-81/032 and 033, Federal Highway Administration, McLean, VA, 1981.

6. Bergen, A.T. and Monismith, C.L. "Some Fatigue Considerations in the Design of Asphalt Pavements." Proc. Canadian Technical Asphalt Association, Vol. 17, 1972.

7. Hveem, F.F. "Pavement Deflections and Fatigue Failures." Highway Research Board Bulletin 114, Highway Research Board, Washington, D.C., 1955.

8. Tholen, O., Sharma, J. and Terrel, R.L.. "Comparison of Falling Weight Deflectometer with Other Deflection Testing Devices." Transp. Resrch. Rec. 1007, Transp Resrch Board, Washington, D.C., 20-26.

9. Ulliditz, P. Pavement Analysis. Elsvier, New York, N.Y., 1987

10. Hoffman, M.S. and Thompson, M.R. "A Comparative Study of Selected Non-destructive Testing Devices." Transp. Resrch. Rec. No. 852, Transp. Resrch. Board, Washington, D.C., 1982, 32-42.

11. Hossain, A.S.M.M., Deflection Analysis of Flexible Pavements Using Nondestructive Test Data, Ph.D. Dissertation, Dept. of Civil Engineering, Arizona State University, Tempe, July, 1990.

12. Painter, L.J., CHEVRON N-layer Program - Improved Accuracy, California Research Corporation, Richmond, CA, 1980.

13. Michelow, J. Analysis of Stresses and Displacements in a N-layered Elastic System Under a Uniformly Distributed Load On a Circular Area, California Research Corporation, Richmond, California, 1963.

14. Jung, F.W.. "Interpretation of Deflection Basin for Real-world Layer Materials of Flexible Pavements." Paper Presented at the 69th Annual Transp. Resrch. Board Meeting, Washington, D.C., 1990.

15. AASHTO, American Association of State Highway and Transportation Official, Guide for Design of Pavement Structures, Washington, D.C, 1986.

16. ARE Inc., Asphalt Concrete Overlays of Flexible Pavements, Report No. FHWA A-RD-75-76, Vol. II, Austin, TX, June, 1976.

17. Karan, M.A., Haas, R. and Walker, T., "Illustration of Pavement Management: From Data Inventory to Priority Analysis", TRR No. 814, TRB, Washington, D.C., 1981.

18. Koole, R.C., " Overlay Design Based on Falling Weight Deflectometer Measurements", TRR No. 700, TRB, Washington, D.C., 1979.

19. Way, G.B., Eisenberg, J.F. and Delton, J.P., Arizona's Pavement Management System, Phase II: Analysis of Testing Frequency for Pavement Evaluation, Report No. FHWA/AZ-81/169-1.

20. Classen, A.I.M. and Ditmarsch, R., "Pavement Evaluation and Overlay Design." Proc. 4th Intl. Confc. on Structural Design of Asphalt Pavements, Vol. 1, University of Michigan, Ann Arbor, 1977, pp. 649-661.

21. Lytton, R.L., Roberts, F.L. and Stoffels, S., Determination of Asphaltic Concrete Pavement Structural Properties by Nondestructive Testing, Final Report, NCHRP, Feb., 1990.

22. Montgomery, D.C., Design and Analysis of Experiments, 2nd Edn, John Wiley & Sons, New York, N.Y., 1984.

23. Rwebangira, T., Hicks, R.G., and Truebe, M., "Sensitivity Analysis of Selected Backcalculation Procedures.". TRR No. 1117, TRB, Washington, D.C., 1987.

24. Irwin, L.H., Yang, W.S., and Stubstad, R.N., "Deflection Reading Accuarcy and Layer Thickness Accuracy in Backcalculation of Pavement Layer Moduli", STP 1026, ASTM, Philadelphia, PA, 1989, pp.229-244.

25. Anderson, V.L. and McLean, R.A., Design of Experiments: A realistic Approach, Marcel Dekker Inc., New York, N.Y., 1974

APPENDIX A

TABLE A.1- Spatial Variabilty of Sensor Readings at Different Sites

Site/Sensor	Mean Deflection (mils)	Standard Deviation (mils)	C.V. (%)	Avg.C.V. for all (%)
1/1	15.32	4.02	26.3	
2	9.67	2.43	25.1	
3	4.92	1.25	25.5	
4	2.49	0.74	29.6	31.5
5	1.44	0.51	35.4	
6	0.99	0.41	41.7	
7	0.70	0.26	36.9	
2/1	8.74	1.31	15.0	
2	6.65	0.82	12.4	
3	4.74	0.47	9.90	
4	3.44	0.35	10.1	10.9
5	2.63	0.24	9.20	
6	2.08	0.20	9.80	
7	1.70	0.17	9.90	
3/1	8.98	0.79	8.80	
2	6.58	0.74	11.2	
3	4.64	0.49	10.60	
4	3.46	0.36	10.5	9.56
5	2.69	0.25	9.40	
6	2.18	0.18	8.30	
7	1.80	0.15	8.10	
4/1	8.39	0.42	5.00	
2	6.75	0.39	5.70	
3	5.10	0.29	5.60	
4	3.77	0.26	7.00	6.93
5	2.74	0.17	6.20	
6	2.01	0.17	8.30	
7	1.45	0.16	10.7	
5/1	6.74	0.19	2.80	
2	5.76	0.18	3.20	
3	4.48	0.25	5.60	
4	3.29	0.29	8.90	10.1
5	2.33	0.28	12.1	
6	1.66	0.26	15.9	
7	1.11	0.25	22.2	

TABLE A.1 (Continued)

Site/Sensor	Mean Deflection (mils)	Standard Deviation (mils)	C.V. (%)	Avg.C.V. for all (%)
6/1	15.37	2.50	16.3	
2	11.39	1.77	15.5	
3	7.62	0.93	12.2	
4	5.19	0.54	10.5	10.97
5	3.73	0.28	7.40	
6	2.85	0.22	7.60	
7	2.34	0.17	7.30	
7/1	12.16	0.45	3.70	
2	9.08	0.32	3.50	
3	5.96	0.21	3.50	
4	3.67	0.17	4.70	7.4
5	2.25	0.15	6.70	
6	1.44	0.17	11.7	
7	0.98	0.18	18.1	
8/1	18.93	6.13	32.4	
2	13.71	3.23	23.5	
3	8.09	1.22	15.1	
4	4.96	0.71	14.4	18.9
5	3.22	0.39	12.1	
6	2.36	0.38	16.2	
7	1.80	0.33	18.5	
9/1	16.96	1.40	8.20	
2	11.23	0.70	6.20	
3	6.20	0.29	4.70	
4	3.32	0.13	3.90	7.5
5	1.91	0.16	8.20	
6	1.31	0.10	7.90	
7	0.98	0.13	13.7	
10/1	26.70	6.14	23.0	
2	6.31	1.34	21.2	
3	2.01	0.23	11.6	
4	1.18	0.10	8.90	12.5
5	0.92	0.07	8.10	
6	0.72	0.05	7.60	
7	0.57	0.04	7.00	
11/1	13.98	1.62	11.6	
2	6.47	0.76	11.7	
3	2.35	0.40	17.0	
4	1.25	0.16	13.1	11.1
5	0.92	0.08	8.20	
6	0.74	0.07	8.70	
7	0.61	0.05	7.40	

TABLE A.1 (Continued)

Site/Sensor	Mean Deflection (mils)	Standard Deviation (mils)	C.V. (%)	Avg.C.V. for all (%)
12/1	18.29	1.66	9.10	
2	13.31	1.29	9.70	
3	8.57	0.91	10.6	
4	5.45	0.61	11.2	11.2
5	3.52	0.41	11.7	
6	2.54	0.31	12.2	
7	2.00	0.275	13.7	
13/1	12.28	2.00	16.2	
2	6.91	1.09	15.8	
3	3.05	0.52	16.9	
4	1.48	0.23	15.3	13.84
5	0.91	0.11	11.9	
6	0.68	0.07	10.4	
7	0.55	0.06	10.4	
14/1	9.56	2.84	29.7	
2	6.81	2.17	31.8	
3	3.61	1.22	33.7	
4	1.99	0.70	34.9	35.44
5	1.19	0.46	38.3	
6	0.82	0.31	38.1	
7	0.58	0.24	41.6	
15/1	6.62	2.54	38.4	
2	4.62	2.00	43.3	
3	2.57	1.30	50.6	
4	1.48	0.83	56.0	49.30
5	0.94	0.54	57.0	
6	0.68	0.35	52.0	
7	0.52	0.25	47.6	
16/1	13.71	3.64	26.6	
2	5.01	1.42	28.3	
3	2.33	0.76	32.5	
4	1.47	0.48	32.5	32.20
5	1.10	0.37	33.6	
6	0.88	0.32	36.4	
7	0.72	0.26	35.5	

TABLE A.2- Correlation among Layer Moduli and Number of 18KESALs for Weak Pavement

	E_{AC}	E_{AB}	E_{SM}	NO. OF 18KESALs
E_{AC}	1.0	0.0	0.0	0.500*
E_{AB}	0.0	1.0	0.0	0.735*
E_{SM}	0.0	0.0	1.0	0.328*
NO. OF 18KESALs	0.5*	0.735*	0.328*	1.0

* Significant at $\alpha = 10\%$

TABLE A.3- Correlation among Layer Moduli and Number of 18KESALs for Medium Stiff Pavement

	E_{AC}	E_{AB}	E_{SM}	NO. OF 18KESALs
E_{AC}	1.0	0.0	0.0	0.746*
E_{AB}	0.0	1.0	0.0	0.490*
E_{SM}	0.0	0.0	1.0	0.300*
NO. OF 18KESALs	0.746*	0.490*	0.300*	1.0

* Significant at $\alpha = 10\%$

TABLE A.4- Correlation among Layer Moduli and Number of 18KESALs for Stiff Pavement

	E_{AC}	E_{AB}	E_{SM}	NO. OF 18KESALs
E_{AC}	1.0	0.0	0.0	0.868*
E_{AB}	0.0	1.0	0.0	0.319*
E_{SM}	0.0	0.0	1.0	0.226*
NO. OF 18KESALs	0.868*	0.319*	0.226*	1.0

* Significant at $\alpha = 10\%$

TABLE A.5- Spatial Variability of Backcalculated Layer Moduli

Site	Layer	Mean (ksi)	St. Dev. (ksi)	C.V. (%)	Avg. CV (%)	Range Min.	Max.
1	AC	155	79.5	50		110	343
	BS	66	10.5	16		60	87
	AB	24	11.8	49	29	10	38
	SM	11	1.3	12		10	14
	SG	14	2.4	16		12	19
2	AC	200	51.5	26		125	253
	BTB	205	83	40	35	152	433
	SM	21	10.9	51		10	46
	SG	19	4	21		12	23
3	AC	217	28.3	13		186	274
	BTB	189	14.3	7.5	15	171	224
	SM	37	10.8	29		13	46
	SG	19	1.8	9		17	23
4	AC	389	60	15		362	516
	CTB	438	143	33	15	254	692
	SM	29	1.4	5		27	31
	SG	12	0.81	7		11	13
5	AC	349	52	15		287	390
	AB	65	2.7	41	18	61	69
	SM	34	3.0	9		27	38
	SG	14	1.1	7.5		13	15
6	AC	142	65	46		64	271
	CTB	87	67	76	46	60	275
	SM	13	5	42		10	24
	SG	13	3	20		10	14
7	AC	419	11	31		402	436
	BS	101	21	20		77	125
	SM	17	2.0	12	19	15	20
	SG	13	1.4	14		10	15
8	AC	180	100	56		41	302
	AB	38	13	34	32	12	48
	SM	15	4	25		10	23
	SG	9	1.2	14		7	10
9	AC	237	56	23		126	309
	AB	32	14	45	23	17	58
	SM	14	2.5	18		12	18
	SG	14	0.8	6		13	16

TABLE- A.5 (Continued)

Site	Layer	Mean (ksi)	St. Dev. (ksi)	C.V. (%)	Avg. CV (%)	Range Min.	Max.
10	AC	265	23	9		233	279
	BS	270	38	14	24	230	358
	SM	12	6	50		10	27
	SG	48	10	21		32	60
11	AC	227	38	17		181	307
	AB	66	25	38	29	33	96
	SM	26	11	42		15	52
	SG	47	9	18		31	60
12	AC	199	63	32		141	332
	BS	60	25	42		20	94
	AB	18	13	74	43	10	41
	SM	15	7	44		10	24
	SG	12	3	25		9	16
13	AC	88	26	30		54	125
	AB	31	18	58	45	10	56
	SM	22	13	60		10	55
	SG	44	14	32		26	60
14	AC	1028*	194	19		814	1371
	AB	80	13	16	22	62	1002
	SM	41	17	42		10	78
	SG	32	13	40		19	60
15	AC	1229*	160	13		1059	1523
	AB	77	8	10	22	56	85
	SM	28	15	54		10	48
	SG	24	2.5	10		21	26
16	AC	232*	82	35		175	423
	AB	49	20	40	34	30	91
	SM	38	15	39		22	72
	SG	37	7	20		29	53

* Uncorrected Modulus

Note: AC: Asphalt Concrete, BS: Bituminous Surface, BTB: Bituminous Treated Base, CTB: Cement Treated Base, AB: Aggregate Base, SM: Select Material

APPENDIX B

TABLE B.1- Summary Statistics for Backcalculatedion of Asphalt Concrete Modulus (in ksi)

Site	10 Tests		5 Tests		3 Tests		1 Test
	Mean	S.D.	Mean	S.D.	Mean	S.D.	Mean
1	155	80	123	13	115	0.9	116
5	349	52	374	22	367	20	354
6	142	65	127	54	120	85	79
7	419	11	425	9.5	421	4	424
8	180	100	165	98	123	97	229
9	237	56	232	77	207	29	239
11	227	38	239	44	229	25	252
12	199	63	218	79	254	75	332
13	89	26	87	22	72	21	95

TABLE B.2- Summary Statistics of the Calculated Number of 18 Kip ESALs (in millions)

Site	10 Tests		5 Tests		3 Tests		1 Test
	Mean	S.D.	Mean	S.D.	Mean	S.D.	Mean
1	11.6	12	6.3	2.35	4.2	0.11	4.3
5	202	35	218	13.4	214	17.1	220.8
6	44	32	41	42	57.5	55.5	24.5
7	80	28	73	16	65	13	53
8	7	6	5	5.6	2.9	4.1	7.6
9	5.3	2.7	5.5	2.9	4.7	3.9	9.1
11	11.4	8.2	9.2	6.6	16.5	9.1	19.5
12	3.7	2.2	3.8	1.84	5.5	2.4	3.5
13	10.2	11.8	13.2	15.7	16	20.2	39.3

TABLE B.3- Summary of Linear Regression Parameters

Parameter	Test	N	A	B	R^2	SEE
EAC	X10 Vs. X5	9	23.5	0.897	0.98	14.9
	X10 Vs. X3	9	44.97	0.834	0.93	28.9
	X10 Vs. X1	9	42.64	0.761	0.79	50.6
N18	X10 Vs. X5	9	1.98	0.93	0.995	5.04
	X10 Vs. X3	9	0.98	0.95	0.98	8.99
	X10 Vs. X1	9	2.73	0.91	0.94	17.54

TABLE B.4- ANOVA for Regression Analysis

Parameter	Source of Variation	Sum Sq.	dof	Mean Sq.	F-ratio
	Due to Regr.	68171.5	1	68171.5	26.67*
EAC	About Regr.	17887.4	7	2555.3	
	Total	86058.9	8		
	Due to Regr.	31850.3	1	31850.3	103.5*
N18	About Regr.	2153.60	7	307.7	
	Total	34003.9	8		

* significant at 5% level of significance

TABLE B.5- Paired t-test Results

Parameter	Diff. in Mean	dof	t-statistic	Results*
EAC	-13.67	8	-0.74	Not Signif.
N18	-0.910	8	-0.16	Not Signif.

* at 5% level of significance

TABLE B.6- Effect of Testing Frequency on the Variation of Structural Capacity

Site	No. of Tests/ Mile	N18 (millions)								
		Run 1			Run 2			Run 3		
		X	σ	CV (%)	X	σ	CV (%)	X	σ	CV (%)
14	10	1862	810	44	1862	810	44	1862	810	44
	7	1858	805	43	2180	686	32	2179	688	32
	5	2184	826	38	2126	904	43	1598	696	44
	3	1574	1155	73	2729	184	7	2061	1008	49
15	10	1197	275	23	1197	275	23	1197	275	23
	7	1170	292	25	1159	300	26	1232	306	25
	5	1008	195	19	1205	332	28	1166	300	26
	3	1460	134	9	1450	145	10	1283	408	32
16	10	18	19	106	18	19	106	18	19	106
	7	18	23	128	18	23	128	17	23	135
	5	26	25	96	15	8	53	10	5	50
	3	31	33	107	8	3	38	13	5	39

TABLE B.7- Results of Kolmogorov-Smirnov (K-S) tests for 18KESALs

Site	N	K-S D-statistic	Dcritical	Results
14	10	0.17	0.37*	Not Significant
15	10	0.17	0.37*	Not Significant
16	10	0.20	0.37*	Not Significant

* at 5% level of significance

TABLE B.8- Results of t-tests

Site	Run	Test	t-statistic	dof	t-critical	Results
14	1	X10 Vs. X7	0.01	15	± 1.75	Not Sig.
	2	"	-0.84	"	"	"
	3	"	-0.84	"	"	"
	1	X10 Vs. X5	-0.72	13	± 1.77	Not Sig.
	2	"	-0.57	"	"	"
	3	"	-0.62	"	"	"
	1	X10 Vs. X3	0.50	11	± 1.79	Not Sig.
	2	"	-1.78	"	"	"
	3	"	-0.36	"	"	"
15	1	X10 Vs. X7	0.19	15	± 1.75	Not Sig.
	2	"	0.27	"		
	3	"	-0.25	"	"	"
	1	X10 Vs. X5	1.36	13	± 1.77	Not Sig.
	2	"	-0.05	"	"	"
	3	"	0.20	"	"	"
	1	X10 Vs. X3	-1.56	11	± 1.79	Not Sig.
	2	"	-1.50	"	"	"
	3	"	-0.43	"	"	"
16	1	X10 Vs. X7	.001	15	± 1.75	Not Sig.
	2	"	-0.01	"	"	"
	3	"	-0.11	"	"	"
	1	X10 Vs. X5	-0.68	13	± 1.77	Not Sig.
	2	"	0.37	"	"	"
	3	"	0.90	"	"	"
	1	X10 Vs. X3	-0.92	11	± 1.79	Not Sig.
	2	"	0.87	"	"	"
	3	"	0.46	"	"	"

TABLE B.9- Results of t-tests

Site	Run	Test	t-statistic	dof	t-critical	Results
16	1	X7 Vs. X5	-0.68	10	±1.81	Not Sig.
	2	X7 Vs. X5	0.12	10	±1.81	Not Sig.
	3	X7 Vs. X5	1.43	10	±1.81	Not Sig.
	1	X5 Vs. X3	0.88	6	±1.94	Not Sig.
	2	X5 Vs. X3	1.10	6	±1.94	Not Sig.
	3	X5 Vs. X3	0.78	6	±1.94	Not Sig.
15	1	X7 Vs. X5	1.07	10	±1.81	Not Sig.
	2	X7 Vs. X5	-0.25	10	±1.81	Not Sig.
	3	X7 Vs. X5	0.37	10	±1.81	Not Sig.
	1	X5 Vs. X3	-3.48	6	±1.94	Signif.
	2	X5 Vs. X3	1.18	6	±1.94	Not Sig.
	3	X5 Vs. X3	-0.47	6	±1.94	Not Sig.
14	1	X7 Vs. X5	-0.57	10	±1.81	Not Sig.
	2	X7 Vs. X5	0.29	10	±1.81	Not Sig.
	3	X7 Vs. X5	0.63	10	±1.81	Not Sig.
	1	X5 Vs. X3	-0.27	6	±1.94	Not Sig.
	2	X5 Vs. X3	-1.34	6	±1.94	Not Sig.
	3	X5 Vs. X3	0.74	6	±1.94	Not Sig.

* at 5% level of significance.

TABLE B.10- Results of Kolmogorov-Smirnov Tests for A

Site	N	K-S D-stat.	Dcritical	Results
16	10	0.14	0.41* 0.37**	Not Significant
14	10	0.22	0.41* 0.37**	Not Significant
15	10	0.18	0.41* 0.37**	Not Significant

* at level of significance = 10%
** at level of significance = 5%

TABLE B-11- Results of t-tests for A

Site	Test	t-stat.	dof	t-critical*	Results
16	X10 Vs. X7	-0.50	15	±1.75	Not Sig.
	X10 Vs. X5	-0.07	13	±1.77	Not Sig.
	X10 Vs. X3	0.30	11	±1.79	Not Sig.
	X07 Vs. X5	0.36	10	±1.81	Not Sig.
	X05 Vs. X3	-0.30	6	±1.94	Not Sig.
15	X10 Vs. X7	0.67	15	±1.75	Not Sig.
	X10 Vs. X5	-0.67	13	±1.77	Not Sig.
	X10 Vs. X3	1.41	11	±1.79	Not Sig.
	X07 Vs. X5	-1.31	10	±1.81	Not Sig.
	X05 Vs. X3	-2.08	6	±1.94	Signif.
14	X10 Vs. X7	0.09	15	±1.75	Not Sig.
	X10 Vs. X5	0.17	13	±1.77	Not Sig.
	X10 Vs. X3	-0.21	11	±1.79	Not Sig.
	X07 Vs. X5	0.08	10	±1.81	Not Sig.
	X05 Vs. X3	0.29	6	±1.94	Not Sig.

* at level of significance = 5%